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Flexural behavior of steel-concrete composite beams with U-shaped steel girders

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Abstract

This paper focuses on a new type of steel-concrete composite beams consisting of U-shaped steel girders and angle connectors. Compared with conventional composite beams consisting of wide flange girders and headed stud connectors (or short channel connectors), the composite beams considered in this study have favorable flexural performance while reducing the excessive costs and potential construction challenges due to installation of the stud and/or channel shear connectors. Through four-point bending tests on five specimens, this research team experimentally investigated flexural behavior of such new composite beams. The five specimens were varied to have different angle connector intervals and installation locations. Test results showed that composite beams with angle connectors welded to the webs of U-shaped steel girder failed in brittle failure modes while composite beams with angle connectors welded on the top flange of U-shaped steel girder failed in ductile failure mode. Moreover, finite element analysis were performed and the results were verified by the experimental results. According to the parametric analysis results, concrete strength has little effect on flexural behavior of composite beams while increasing yield stress of steel girder could significantly increase the flexural resistance but could not change the initial stiffness. Increasing the height of steel girder, the thickness and width of bottom flange are recommended to improve the flexural behavior of composite beams.

Keywords: *Composite beams; U-shaped steel girder; angle connectors; flexural behavior; experimental investigation; finite element analysis.*

1. Introduction

Steel-concrete composite beams (referred to herein as composite beams) possess the advantages of both structural steel and reinforced concrete. The advantages of composite beams have been long recognized [1] and application of composite beams can be found in many buildings and bridges [2]. Nevertheless, design challenges still exist, hindering the widespread acceptance of such viable structural components.

For example, the most popular shear connectors in composite beams are headed studs and short channels [3, 4]. However, installation of headed studs requires special automatic tools and skilled operators, for which additional cost may be incurred, offsetting the other advantages of composite beams. Although adoption of short channels can help reduce the cost associated with connector installation, the channel connectors may congest the longitudinal rebars in the concrete slab, resulting in construction difficulties. Therefore, there is a research need to develop alternative shear connectors for composite beams. Moreover, application of composite beams may be impeded in some cases due to the insufficient torsional resistance, e.g., in the exterior beam in a building floor system or a bridge deck system supports the tributary gravity load transferred from one side of the beam [5]. To resist the torsional demand, a wide flange girder much larger than the one designed for the bending moment demand may have to be used in some cases, leading to a less economical composite beam design. Note that torsional resistance of the composite beam with a wide flange girder is generally lower than the one with a box girder [6]. As such, it is necessary to develop alternative girders with torsional performance similar to or equal to that of a box girder for composite beams.

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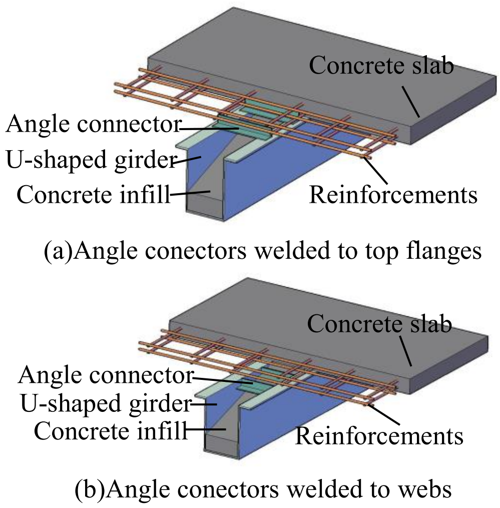

Fig.1. Composite beams in this research.

This research team explored a practical solution to the above issue, in which a U-shaped steel girder was used as an alternative to the steel girder and equal-leg angles welded along the transverse direction to the girder were used as the shear connectors. Fig. 1 schematically shows the system considered in this investigation. As shown, the angles can be welded either to the top flanges or to the webs of the U-shaped girder. Conceptually, the vertical legs of the angles can resist the longitudinal shear force while the horizontal legs can mitigate the separation of concrete deck from the steel girder. Moreover, the angles help reinforce the U-shaped girder, enabling the girder to exhibit a superior torsional moment resistance. Although the infill concrete in the U-shaped girder increases the weight of the girder, it helps mitigate local buckling of the webs of the girder and improve strength and stiffness of the girder. The infill concrete can also enhance fire resistance of the composite beam as it does in other types of composite members. Therefore, it is necessary to study its flexural behavior to promote its application.

The objective of this research was to evaluate flexural behavior of such composite beams through experimental studies and finite element analysis. The results obtained from this investigation could form a basis for a better understanding of the fundamental behavior of such composite beams and help promote their applications in future constructions.

2. Specimen design and construction

Five composite beams, designated as Specimens CB1 to CB5, were tested under 4-point bending. All the specimens were identical except that their shear connectors (including connector type, installation location and connector interval) were varied. Fig. 2 presented the configurations and dimensions of specimens in the test. As illustrated in Fig. 2b, two layers of steel rebars with nominal diameter of 10 mm were spaced at 200 mm and 250 mm along the transverse and longitudinal directions to reinforce the concrete slab, respectively. Steel angles with equal legs (50-mm wide and 5-mm thick) were used as shear connectors in all specimens. It is recognized that no analytical models were readily available for calculating the shear force transfer capacity of each angle connector at the time of the investigation. Alternatively, the formula for calculating shear force transfer capacity of short channel connectors in composite beams in the Chinese Code for Design of Steel Structures [7] was adopted to predict the shear strength of angle connectors in this research. To quantitatively compare the capacity of a group of connectors in transferring the shear force between the concrete slab and the steel girder, the shear transfer coefficient, k_s, was calculated as the ratio of the available strength of the connectors between the point of zero moment and the maximum moment to the required strength of shear connectors to make the beam fully composite. Then the shear connectors for different specimens were designed and listed in Table 1 according to the nominal compressive strength of concrete (30 MPa) and yield strength of steel (345 MPa). Material properties of the steel elements and concrete used in the specimens were evaluated through tests of material samples. Table 2 presents yield strength, f_y, ultimate strength, f_u, modulus of elasticity, E_s, and Poisson's ratio, v of each type of steel. Table 3 reports the concrete compressive strength associated with cubes and prisms, f_c and f_c', modulus of elasticity, E_c for each specimen. The nominal yield and ultimate tensile strengths of the steel in the headed shear studs are 322 MPa and 410 MPa, respectively.

Table 1. Design of shear connectors.

Specimen	Numbers	location	Nominal k_s
CB1	18	Web	1.00
CB2	14	Web	0.77
CB3	10	Web	0.55
CB4*	14	Web	1.00
CB5	14	Flange	0.77

*a pair of headed studs of Φ16 × 80 (diameter x height; unit: mm) were also provided every 600 mm.

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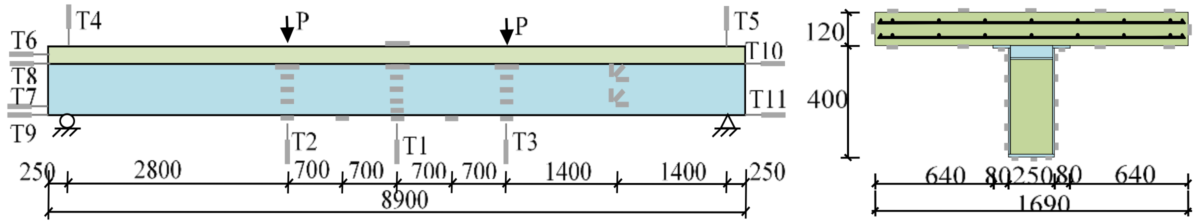

(a) Elevation (b) Cross-section

Fig. 2. Geometries and instrumentation of tested specimens (unit: mm).

Table 2. Properties of steel.

Type	t /mm	f_y /MPa	f_u /MPa	E_s /GPa	v
Flange	9.57	349.5	514.7	207	0.32
Web	5.60	380.8	547.2	196	0.31
Angle steel	4.44	346.6	476.4	212	0.32
rebar	Φ10	395.8	541.0	216	-[a]

[a] Not available.

Table 3. Properties of concrete.

Parameter	CB1	CB2	CB3	CB4	CB5
f_{cu} (MPa)	39.5	35.9	38.8	34.8	42.6
f_c' (MPa)	30.0	27.3	29.5	26.4	32.4
E_c (GPa)	29.0	30.8	26.8	28.3	32.4

3. Test setup and loading scheme

Four-point bending tests were performed for all the specimens. Fig. 3 schematically shows the test setup. As shown, each specimen was simply supported at the ends and two identical point loads were applied at its one-third points. Load on each specimen was monotonically increased through the force control protocol during the test. The load increment was selected to be 40 kN per step at speed of 40 kN/min up to the elastic limit of each specimen. Then, the load increment reduced to 20 kN per step till the ultimate state. The load was sustained for 3 minutes at the end of each loading step to allow observation and recording the progressively developed damages in each specimen. The distributions of the displacement transducers and strain gauges attached to each specimen were presented in Figs. 2 and 3.

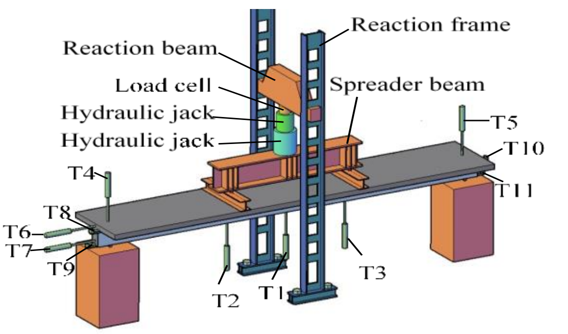

Fig. 3. Test setup.

4. Test results

4.1. Failure mode

Overall, the five specimens exhibited similar crack progressions in their concrete slabs with the increases of external loads; however, their failure modes can be differentiated into two categories. Among all the specimens, Specimens CB4 and CB5 failed by concrete crushing and did not exhibit significant slip deformations between concrete slabs/infills and steel girders as shown in Fig. 4a. Unlike these two specimens, Specimens CB1 to CB3 all developed significant slip deformations between concrete slabs/infills and steel girders under the close-to-ultimate levels of loads as shown in Fig. 4b.

(a) Specimen CB4 (b) Specimen CB1

Fig. 4. Slip deformations at beam ends.

4.2. Load-deflection curves

Fig. 5 compares development of the deflection recorded at the mid-span of each specimen with the increase of external loads. As shown in Fig. 5, under the load up to 25% of the ultimate strength (around 150 KN), elastic flexural stiffness of the beam remains the same among all the specimens. That is because, during this stage, concrete slab and steel beam behave as a monolithic unit since the angle connectors remains fully elastic. Additionally, the bond between concrete slab and the steel girder can transfer the longitudinal shear force in the initial stage. Beyond the load associated with 25% of the ultimate strength, these specimens exhibit

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significant stiffness degradations due to yielding of the steel girders and deformation of the angle connectors. Stiffness values of these specimens during the unloading process are similar to their initial stiffness and all the specimens exhibit significant residual deformations after removal of the external loads.

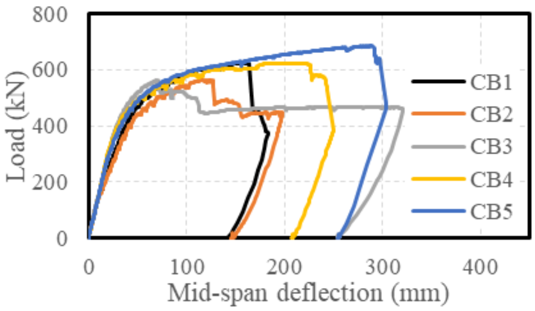

Fig. 5. Load vs. mid-span deflection curves.

Note that Specimens CB1 to CB3 have the reduced degrees of composite action. As indicated by the curves of these three specimens shown in Fig. 8, the higher degree of the composite action tends to increase the mid-span deflection associated with the strength limit in the composite beams consisting of the angle connectors welded to the webs of the U-shaped girders. It is also shown that Specimens CB1 to CB3 are less ductile in comparison with others. Moreover, it is found that Specimen CB4 exhibits a larger mid-span deflection associated with the strength limit compared with Specimens CB1 to CB3, suggesting that inclusion of headed studs can improve deformation capacity and delay occurrence of strength degradation in the composite beams with angle connectors welded to the webs of the U-shaped girders. The observed improvement in deformation capacity is primarily due to the following two aspects: (1) the shear studs have better anchorage in the concrete slab due to their headed ends embedded in the concrete slab; and (2) the shear studs were welded to the flanges of the U-shaped girder, which is a more effective location for shear transfer connectors to enable the composite action (this will be further confirmed by the test results from Specimen CB5). Further, it is found from Fig. 8 that Specimen CB5 which includes angle connectors welded on the top flanges of the U-shaped girder has the highest flexural strength and the largest deflection associated with the strength limit, indicating that angle connector welded on the top flanges of the U-shaped girder is more ductile than other type of connectors considered in this investigation. Moreover, Specimen CB1 and CB5 which are both design to be full shear connection (k_s=1.0) had significantly different behavior in capacity and ductile, showing that the formula recommended in Chinese Code for Design of Steel Structures [7] cannot properly approximate the shear strength of angle connectors in this study.

5. Finite element analysis

Three-dimensional finite element (FE) model has been developed by ABAQUS to study the bending behavior of composite beams with U-shaped steel girder and angle connectors. Geometric and material non-linear behaviors are considered in the model. The shell element S4R is employed for steel girder and angle connectors. The solid element C3D8R is applied for concrete component and truss element T3D2 is employed for reinforcements. Hard contact and Penalty friction are defined in normal and tangential direction between concrete and steel girder, respectively. The friction coefficient is 0.3. Angle connectors and steel girder are merged as a new part while the reinforcements and angle connectors are embedded in concrete slab. Headed studs in CB4 are simulated by tie both nodes in concrete slab and steel girder at the position of headed studs. A typical FE model is shown in Fig. 6. Half model is performed according to the symmetric characteristic and displacement loading is applied to improve the calculation accuracy and efficiency.

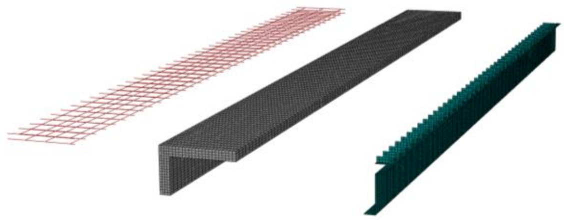

Fig. 6. FE model.

The three-linear stress-strain relationship with consideration of strain hardening is applied for steel, which is shown in Fig. 7a. The strain hardening modulus of steel is 1/100 elastic modulus. The concrete is modelled by using the damage plasticity models in ABAQUS. The non-linear stress-strain relationship in the Chinese Code for Design of Concrete Structures GB 50010-2010 [8] shown in Fig. 7b is applied in FE model. The parameters of concrete dilation angle, flow potential eccentricity, biaxial/uniaxial compressive stress ratio, tensile/compressive meridian ratio of the second

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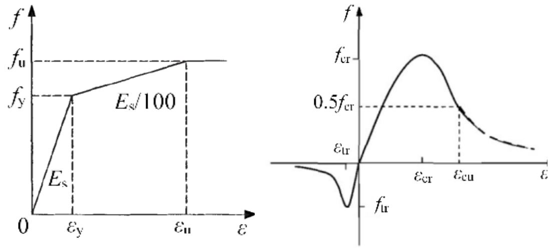
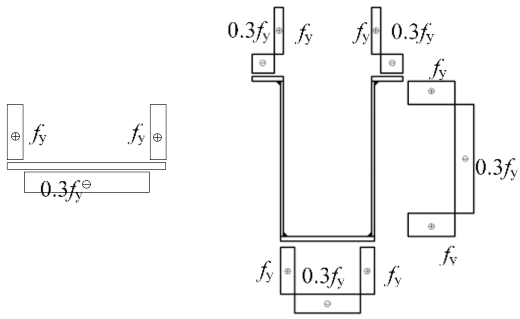
stress invariant and viscosity parameter are respectively set to be 30, 0.1, 1.16, 0.6667 and 0.0005 [9]. The measured values of material properties for steel components and concrete listed in Table 2 and Table 3 are used in the FE analysis.



(a) Steel (b) Concrete

Fig. 7. Stress-strain relationship.

Non-uniform temperature field would generate when welding the steel components and would then lead to residual stress all over the section of steel components after cooling. Welding residual stress would combine with stress produced by external loads and decrease the stiffness of composite beams. The steel girders in this test were welded by several pieces of steel plates, which resulted in large residual stress in the beam section. However, the residual stress would be small in actual applications since the steel girder could be rolled in mass production.



(a) Distribution on steel plate (b) Distribution on U-shaped steel girder

Fig. 8. Distribution of residual stress.

The residual stress distribution on one side of steel tube fabricated by a series of steel plates is simplified as rectangular shape shown in Fig. 8a, which is simplified by measured compressive residual stress [10]. The symbol '+' represents tensile stress while the symbol '–' represents compressive stress. Since the maximum residual stress in compression zone is 0.3 f_y [10], the residual stress distribution on U-shaped steel girder is assumed as Fig. 8b and then the width of tension and compression zone could be obtained by the self-balanced principle. The initial stress can be applied by writing "*INITIAL CONDITIONS, TYPE=STRESS" in input file based on ABAQUS Documentation [9].

The simulated load-deflection curves with and without residual stress are compared in Fig. 9. As shown, residual stress has little influence on the ultimate resistance but have significant effect on the stiffness in the elastic-plastic stage.

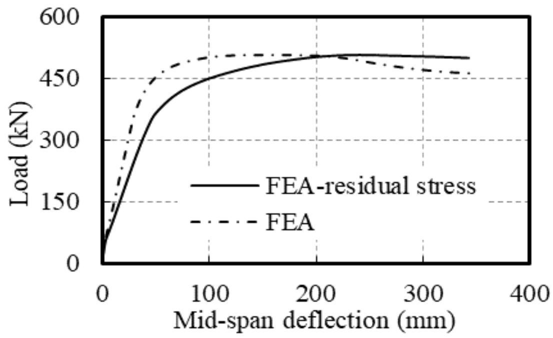
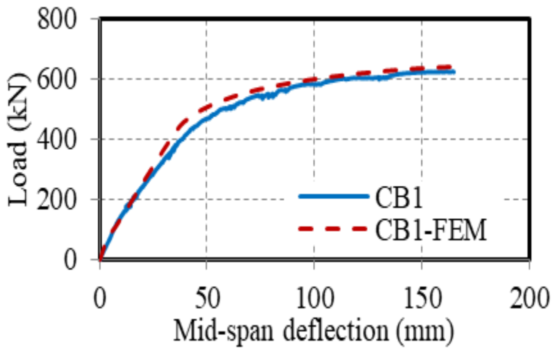
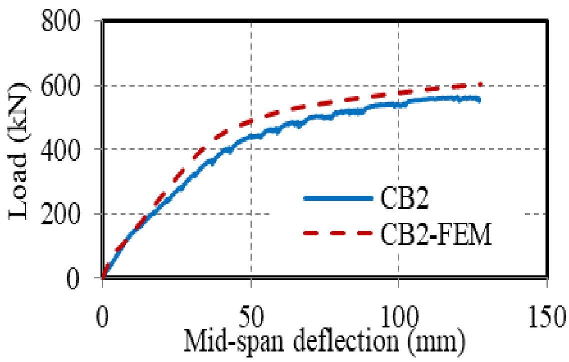


Fig. 9. Influence of residual stress.

The load-deflection curves of all specimens simulated by FE model considering residual stress are compared with tested results in Fig. 10. Good agreement is obtained between two curves, indicating that the FE model with residual stress could simulate the flexural behavior of composite beams with U-shaped steel girder and angle connectors.



(a) CB1



(b) CB2

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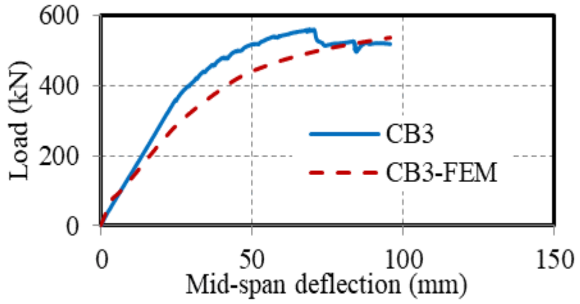
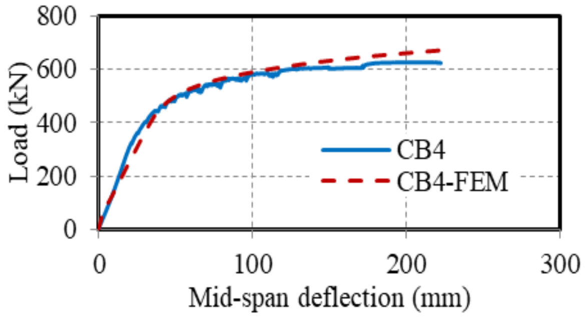
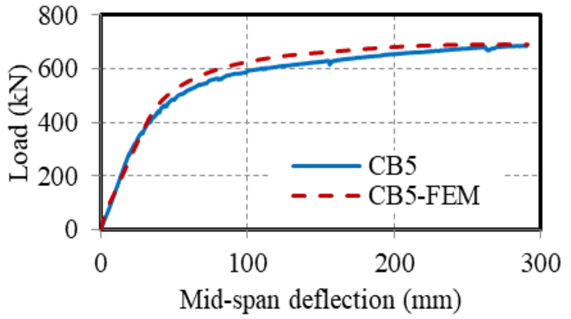


(c) CB3



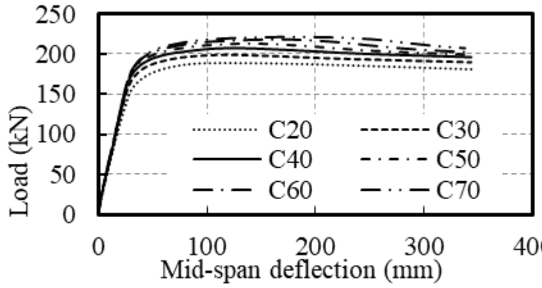
(d) CB4



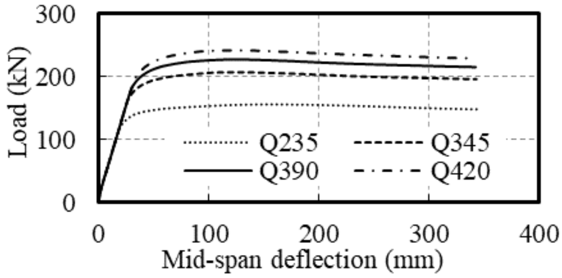
(e) CB5

Fig. 10. Comparison of simulated and tested load-deflection curves.

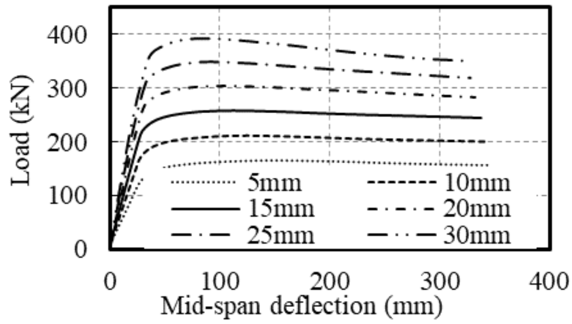
After validating the FE model, parametric analysis is conducted to study the influence of each factor on the flexural behavior of composite beam. Residual stress is not considered herein to focus on the influence of varied parameters. Depth, flange thickness, web thickness, top flange width and bottom flange width of steel girder in specimen selected as the benchmark are 350 mm, 10 mm, 6 mm, 60 mm, 200 mm, while the width and thickness of concrete plate is 1400 mm and 100 mm, respectively. The grades of concrete and steel girder in benchmark are C40 and Q345, respectively. C40 indicates that the standard compressive strength of concrete cube (150 mm × 150 mm × 150 mm) is 40 MPa while Q345 means that the yield stress of steel is 345 MPa. Fig. 11 presented the load-deflection curves of composite beams with different concrete strength, yield stress of steel girder, web thickness, bottom flange thickness, bottom flange width and height of steel girder. As shown, concrete strength has little effect on the flexural behavior of composite beams while increasing the yield stress of steel girder could significantly increase the flexural resistance but could not change the initial stiffness. Meanwhile, increasing the height of steel girder, the thickness and width of bottom flange are recommended to improve the flexural behavior of composite beams.



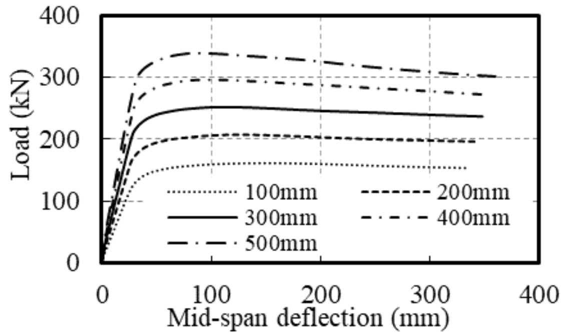
(a) Concrete strength



(b) Yield strength of steel girder



(c) Bottom flange thickness



(d) Bottom flange width

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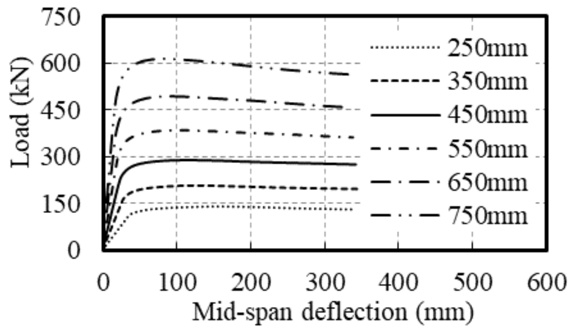

(e) Height of steel girder

Fig. 11. Load-deflection curves of composite beams with different parameters.

6. Conclusions

Based upon the results obtained from this investigation, the following significant conclusions were drawn:

1. Composite beams with angle connectors welded to the webs of the U-shaped steel girders (Specimens CB1 to CB3) exhibit significant slip deformations between concrete and U-shaped steel girders. Replacing part of angle connectors by headed studs is an effective way to improve ductility of composite beams with angle connectors welded to webs.
2. The composite beam with angle connectors welded on the top flange of the U-shape steel girder (i.e., Specimen CB5) exhibit the fully composite action. It fails in a more ductile manner and develops negligible slip deformations between concrete slab/infill and steel girder.
3. FE models were performed to study the flexural behavior of such composite beams and FE model with residual stress could simulate the flexural behavior of composite beams with U-shaped steel girder and angle connectors.
4. Concrete strength has little effect on flexural behavior of composite beams while increasing yield stress of steel girder could significantly increase the flexural resistance but could not change the initial stiffness. Increasing the height of steel girder, the thickness and width of bottom flange are recommended to improve the flexural behavior of composite beams

Acknowledgements

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